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GEOCON

GEOCON PROJECT NO. E9034-04-01

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GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. E9034-04-01
February 2, 2018

Buddy Silvercreek, LLC
6669 Elwood Road
San Jose, California 95120

Attention: Mr. K.D. Patel

Subject: PROPOSED 4-STORY HOTEL
5952 SILVER CREEK VALLEY ROAD
SAN JOSE, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Patel:

In accordance with your authorization, we have performed a geotechnical investigation for the subject hotel development in San Jose, California. Our investigation was performed to observe the soil and geologic conditions that may impact site development for the project as presently planned. The accompanying report presents the results of our investigation and conclusions and recommendations pertaining to the geotechnical aspects of the proposed project. The findings of this study indicate the site is suitable for development as planned provided the recommendations of this report are implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Sincerely,

GEOCON CONSULTANTS, INC.

DRAFT

Jacob Bishop-Moser, EIT
Senior Staff Engineer

DRAFT

Shane Rodacker, PE, GE
Senior Engineer

(1/e-mail) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for a new hotel project at 5952 Silver Creek Valley Road in San Jose. The purpose of this investigation was to evaluate the subsurface soil and geologic conditions in the area of planned development and provide conclusions and recommendations pertaining to the geotechnical aspects of project design and construction, based on the conditions encountered during our study.

The scope of this investigation included field exploration, laboratory testing, engineering analysis and the preparation of this report. Our field exploration was performed on December 27, 2017 and included the advancement of six exploratory borings to maximum depths of approximately 50 feet below existing grade at the site. The locations of the soil borings are depicted on the Site Plan, Figure 2. A detailed discussion of our field investigation and boring logs are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent geotechnical parameters. Appendix B presents the laboratory test results in tabular format and graphical format. Appendix C presents output from our liquefaction analysis.

The opinions expressed herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE CONDITIONS AND PROJECT DESCRIPTION

The site is an approximately 2¼-acre parcel (Santa Clara County APN 678-93-015) on the southeast side of Silver Creek Valley Road approximately 450 to 700 feet southwest of Hellyer Avenue. The site is generally undeveloped with grasses tilled seasonally for fire control. Web-based mapping indicates the ground surface at the site is generally flat with existing grades on the order of 205 to 210 feet MSL. Fills have been placed at the southwestern margin of the site and graded into a berm with an approximately 3:1 (horizontal:vertical) descent toward the center of the site.

Based on the information provided by Buddy Silvercreek, we understand the proposed project will include the construction of a hotel which will occupy the center of the lot. The building will be constructed at-grade with no subterranean levels. The building will be four stories of wood-framed pre-fabricated modular construction with a one-story masonry-walled entryway. Structural loading information was not available at the time of our study; we have made assumptions based on experience with similar projects. The building will be surrounded by an asphalt parking lot with driveways to both Silver Creek Valley Road and the property to the southwest of the site. A trash enclosure will be located at the south corner of the site.

Grading and improvement plans have not been prepared at the time of this report. We assume minor fills on the order of two feet or less in the building pad area and cuts on the order of four feet or less in the fill areas in the southwestern margin of the site to attain design subgrade elevation for the new development. Deeper cuts may be needed in localized areas for elevators pits. Ancillary site improvements such as new pavements, underground utilities and landscaping are also anticipated. The proposed site configuration is depicted on Figure 2.

3. GEOLOGIC SETTING

San Jose is located within the Coast Ranges Geomorphic Province of California, which is characterized by a series of northwest trending mountains and valleys along the north and central coast of California. Topography is controlled by the predominant geological structural trends within the Coast Range that generally consist of northwest trending synclines, anticlines and faulted blocks. The dominant structure is a result of both active northwest trending strike-slip faulting, associated with the San Andreas Fault system, and east-west compression within the province.

The San Andreas Fault (SAF) is a major right-lateral strike-slip fault that extends from the Gulf of California in Mexico to Cape Mendocino in northern California. The SAF forms a portion of the boundary between two tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF but also distributed, to a lesser extent, across several other faults including the Hayward and Calaveras faults. Together, these faults are referred to as the SAF system.

Basement rock west of the SAF is generally granitic, while to the east it consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (205 to 65 million years old). Overlying the basement rocks are Cretaceous (about 140 to 65 million years old) marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted largely because of movement along the SAF system, which has been ongoing for about the last 25 million years, and regional compression during the last about 4 million years. The inland valleys, as well as the structural depression within which San Francisco Bay is located, are filled with unconsolidated to semi-consolidated deposits of Quaternary age (about the last 1.6 million years). Continental deposits (alluvium) consist of unconsolidated to semi-consolidated sand, silt, clay and gravel, while the bay deposits typically consist of soft organic-rich silt and clay (bay mud) or sand.

Available geologic mapping by the California Geological Survey (CGS) indicates the site is near an interface of Holocene age alluvial fan and alluvial fan levee deposits. Artificial fills are present at the southwestern edge of the site. A geologic cross-section is presented as Figure 3.

4. GEOLOGIC HAZARDS

Geologists and seismologists recognize the San Francisco Bay Area as one of the most seismically-active regions in the United States. The significant earthquakes that occur in the Bay Area are associated with crustal movements along well-defined active fault zones that generally trend in a northwesterly direction.

The site and greater Bay Area are seismically dominated by the presence of the active SAF system. Locally, SAF system movement is distributed across a complex series of strike-slip, right lateral parallel and subparallel faults, which include the San Andreas, Hayward and Calaveras faults, among others.

The table below presents approximate distances to active faults in the site vicinity based on web-based mapping by the United States Geological Survey (USGS) and CGS. Site latitude is 37.2585° N, 121.7844° W.

TABLE 4.1
REGIONAL FAULT SUMMARY

Fault Name	Approximate Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Silver Creek	1 ½	6.9
Hayward – Southern Extension	3 ½	6.7
Monte Vista Shannon	5 ½	6.4
Calaveras	6	6.9
Sargent	11	7.0
San Andreas	12 ½	8.0
Zayante-Vergeles (Upper)	15 ½	7.0
Hayward – South	17 ¼	7.3
Zayante-Vergeles (Lower)	18	7.0
Greenville	19	6.9
Hayward – North	15 ½	7.3
Los Positas	24	6.4
Pleasanton	24 ¼	6.6

Faults tabulated above and many others in the Bay Area are sources of potential ground motion. However, earthquakes that might occur on other faults within the northern California area are also potential generators of significant ground motion and could cause ground shaking at the site.

4.1 Surface Fault Rupture

The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards. No active or potentially-active faults are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. By CGS definition, an active fault is one with surface displacement within the last 11,000 years. A potentially-active fault has demonstrated evidence of surface displacement with the past 1.6 million years. Faults that have not moved in the last 1.6 million years are typically considered inactive.

4.2 Ground Shaking

We used the USGS web-based *Unified Hazard Tool* to estimate the peak ground acceleration (PGA) and mean and modal (most probable) magnitude associated with a 2,475-year return period that corresponds to an event with 2 percent chance of exceedance in 50 years. The USGS estimated PGA is 0.78g and the mean magnitude is 6.6 for Seismic Site Class D (V_s30 = 259 m/sec) based on a 2008 model within the application.

While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

4.3 Liquefaction

The site is located within State of California and Santa Clara County Seismic Hazard Zones for liquefaction. Interactive web-based mapping by USGS and CGS indicates the site soils possesses a "moderate" susceptibility to liquefaction. Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with intense earthquakes. Primary factors that trigger liquefaction are: moderate to strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions (shallow groundwater). Due to the increasing overburden pressure with depth, liquefaction of granular soils is generally limited to the upper 50 feet of a soil profile.

We evaluated the potential for liquefaction and resultant settlements at the site using the sampling data from our borings and the methodology of Youd et. al. (2001). Our evaluation incorporated an earthquake moment magnitude (M_w) of 6.6 and a groundwater depth of 16 feet. The selected groundwater depth is the based on the shallowest reading of California Statewide Groundwater Elevation Monitoring System (CASGEM) wells in the vicinity of the site (see list of references). Based on USGS seismic design criteria for 2016 California Building Code (CBC), a ground motion/Peak Ground Acceleration (PGA) of 0.5g is acceptable for design. However, based on prior experience with the City of San Jose, our analysis incorporated a PGA of 0.78g which was obtained from the USGS *Uniform Hazard Tool* application and based on a 2,475-year return period. This return period corresponds to an event with 2% chance of exceedance in 50 years.

Our liquefaction analysis identified potentially liquefiable layers at our Borings B2 and B5. In general, these layers are located more than 30 feet below existing grade at the site. Consequences of liquefaction can include ground surface settlement, ground loss (sand boils) and lateral slope displacements (lateral spreading). For liquefaction-induced sand boils or fissures to occur, pore water pressure induced within liquefied strata must exert enough force to break through overlying, non-liquefiable layers. Based on methodology recommended by Youd and Garris (1995), which advanced original research by Ishihara (1985), a capping layer of non-liquefiable soil can prevent the occurrence of sand boils and fissures. Based on the presence of the non-liquefiable layer that mantles the site (which was observed to be approximately 33½ feet thick in our soil borings) and the depth to significant liquefiable layers, the potential for ground loss due to sand boils or fissures in a seismic event is considered low.

The likely consequence of potential liquefaction at the site is settlement. Our analysis indicates that total ground surface settlements up to approximately 3 inches may result from liquefaction after a seismic event.

Web-based topographic information indicates the ground surface in the general site vicinity slopes at approximately 0.2 to 0.3% to the north. We evaluated the potential for lateral spreading at the site using the sampling data from our borings and the methodology of Zhang et. al. (2004). Our analysis indicates total lateral displacements of approximately 2 feet or less could occur. Special design considerations may be necessary to mitigate the effect of lateral spreading at the site.

We recommend that foundations and ground improvement systems be designed to accommodate approximately 3 inches of total liquefaction-induced settlement and approximately 2 inches of differential seismic settlement across a horizontal distance of 50 feet. Output from our liquefaction analysis is presented in Appendix C.

4.4 Landslides

There are no known landslides near the site, nor is the site in the path of any known or potential landslides. We do not consider the potential for a landslide to be a significant hazard to this project.

4.5 Tsunamis and Seiches

The site is not located within a coastal area. Therefore, tsunamis (seismic sea waves) are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

5. SOIL AND GROUNDWATER CONDITIONS

5.1 Undocumented Fill

Our Boring B6 encountered undocumented fill materials to depths of approximately 4 feet below existing grade. The source of the fill materials is unknown and documentation relative to fill placement was not provided, but the fills appear to be generally derived from local native materials. As observed in our soil boring, the fill materials consisted of very stiff clays with few sands, silts, and gravels.

5.2 Alluvium

Geologic references map an interface of Holocene age alluvial fan and alluvial fan levee deposits at the site. In our soil borings, the deposits were observed as stiff to hard clays with variable amounts of silt and sand and medium dense to very dense sands with variable amounts of clay. We encountered alluvial deposits to the maximum depth explored – approximately 50 feet below existing grade.

5.3 Groundwater

Groundwater was encountered at depths of approximately 23 feet below existing grade in our soil borings B2 and B5. Historic high groundwater levels in the immediate site vicinity are on the order of 16 feet below existing grade based on CASGEM mapping. Actual groundwater levels will fluctuate seasonally and with variations in rainfall, temperature and other factors and may be higher or lower than observed during our study.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No overriding geotechnical constraints were encountered during our investigation that would preclude the project as presently proposed. Primary geotechnical considerations are the presence of undocumented fills, the presence of expansive soils at grade, and the liquefaction potential of the alluvial materials that underlie the site. Based on the assumed structural loading for the development, we anticipate a ground improvement system will be needed to support the proposed 4-story structure.

6.1.2 The undocumented fills in the southwestern margin of the site will require remedial grading. The undocumented fills should be over-excavated to expose competent alluvial deposits. The exposed bottom should then be scarified to a depth of approximately 1 foot, moisture conditioned to at least 2% above optimum moisture and recompacted to at least 90% relative compaction.

6.1.3 This report identifies ground improvement techniques that are considered feasible for the proposed structures and the site soil and geologic conditions. General discussion on various ground improvement systems is presented herein. Other ground improvement or deep foundations may also be feasible depending on the acceptability of vibration and noise from foundation construction, the environmental characteristics of the soils that underlie the site, local agency requirements or other factors.

6.1.4 The project team should review the information provided herein and other non-geotechnical factors, such as export disposal, when selecting foundation type or ground improvement for the project. The design of specialty foundation types or ground improvement systems should be reviewed by Geocon.

6.1.5 As discussed in Section 4.3, the site is susceptible to liquefaction. Our analysis indicates that, if liquefaction were to occur, total ground surface settlements will be on the order of 3 inches. Based on our experience in the area, we recommend the project be designed to accommodate at least 2 inches of seismically-induced settlement over a distance of 50 feet.

6.1.6 The alluvium at the site possesses a moderate to high expansion potential based on our observations and the results of our laboratory testing. Proper soil moisture conditioning, compaction, and surface drainage are recommended to reduce the shrink-swell potential of the site soils. In addition, a layer of non-expansive fill will be required beneath all interior slabs-on-grade and exterior slabs.

6.1.7 Where shallow foundation systems are used for ancillary structures such as screen walls, post-construction settlement due to assumed loading should be on the order of 1 inch or less with differential settlements of approximately ¾ inch across a horizontal distance of 50 feet.

6.1.8 Any changes in the design, location or elevation of the proposed improvements, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

6.1.9 All references to relative compaction and optimum moisture content in this report are based on ASTM D 1557 (latest edition).

6.2 Seismic Design Criteria

6.2.1 We understand that seismic structural design will be performed in accordance with the provisions of the 2016 CBC which is based on the American Society of Civil Engineers (ASCE) publication *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10). We used the USGS web-based application *US Seismic Design Maps* to evaluate site-specific seismic design parameters in accordance with the 2016 CBC and ASCE 7-10. Results are summarized in Table 6.2.1. The values presented are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 6.2.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC / ASCE 7-10 Reference
Site Class	D	Section 1613.3.2/ Table 20.3-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.5g	Figure 1613.3.1(1) / Figure 22-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.6g	Figure 1613.3.1(2) / Figure 22-2
Site Coefficient, F_A	1.0	Table 1613.3.3(1) / Table 11.4-1
Site Coefficient, F_V	1.5	Table 1613.3.3(2) / Table 11.4-2
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.5g	Eq. 16-37 / Eq. 11.4-1
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.9g	Eq. 16-38 / Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.0g	Eq. 16-39 / Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.6g	Eq. 16-40 / Eq. 11.4-4

6.2.2 Table 6.2.2 presents additional seismic design parameters for projects with Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.2.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.504g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.504g	Section 11.8.3 (Eq. 11.8-1)

6.2.3 Conformance to the criteria presented in Tables 6.2.1 and 6.2.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid structural damage, since such design may be economically prohibitive.

6.3 Soil and Excavation Characteristics

6.3.1 The onsite soils can be excavated with moderate effort using conventional excavation or drilling equipment. We do not anticipate excavations in the native alluvium at the site will generate oversize material (greater than 6 inches in nominal dimension).

6.3.2 Unknown or unanticipated conditions may exist, especially within areas of artificial fill. The artificial fills at the site are undocumented and may contain constituents not reported herein.

6.3.3 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupational Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

6.3.4 Some of the existing fill materials and alluvial soils encountered at the site should be considered “expansive” as defined by 2016 CBC. The recommendations of this report assume proposed foundation systems will derive support in competent engineered fills and/or competent alluvial soils.

6.4 Materials for Fill

6.4.1 Soils generated from cut operations or foundation excavations at the site are suitable for use as engineered fill in structural areas provided they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension. Excavated soils may be wet and require drying prior to use and engineered fill.

6.4.2 Import or low-expansive fill material should be primarily granular with a “low” expansion potential (Expansion Index less than 50), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock larger than 6 inches in greatest dimension.

6.4.3 Environmental characteristics and corrosion potential of import soil materials may also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to its transportation to the site.

6.5 Grading

6.5.1 All clearing operations and earthwork (including over-excavation, scarification, and recompaction) should be observed and all fills tested for recommended compaction and moisture content by representatives of Geocon.

6.5.2 Structural areas should be considered as areas extending a minimum of 5 feet horizontally from a foundation or beyond the outside dimensions of buildings, including footings and overhangs carrying structural loads, and where not restricted by property boundaries.

6.5.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

6.5.4 After complete demolition and removal of existing structures, site preparation should commence with the removal of all existing improvements from the area to be developed/graded. All active or inactive utilities within the construction area should be protected, relocated, or abandoned. Any pipelines to be abandoned that are greater than 2 inches and less than 18 inches in diameter should be removed or filled with sand-cement slurry. Utilities larger than 18 inches in diameter should be removed. Excavations or depressions resulting from demolition and site clearing operations, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.5.5 The undocumented fills in the southwestern margin of the site will require remedial grading. The undocumented fills should be over-excavated to expose competent alluvial deposits. The exposed bottom should then be scarified to a depth of approximately 1 foot, moisture conditioned to at least 2% above optimum moisture and recompacted to at least 90% relative compaction.

6.5.6 Existing soils in building pad areas should be over-excavated one foot below existing grade or one foot below foundation and footing bottoms, whichever is deeper. The resultant over-excavation bottoms should then be scarified to a depth of approximately 1 foot, moisture conditioned to at least 2% above optimum moisture and recompacted to at least 90% relative compaction.

6.5.7 Existing soils in pavement and exterior flatwork areas should be scarified to a depth of approximately 1 foot, moisture conditioned to at least 2% above optimum moisture and recompacted to at least 90% relative compaction.

6.5.8 In general, over-excavated materials may be used for new engineered fill provided they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension. Over-excavations and the exposed bottom surfaces and bottom processing should be observed by our representatives. Supplemental recommendations may be provided based on site conditions during grading. Areas of deeper over-excavation may be required.

6.5.9 All structural fill and backfill should be placed in layers no thicker than will allow for adequate bonding and compaction (typically 8 to 12 inches). Fill soils should be placed and compacted to at least 90% relative compaction at least 2% above optimum moisture content (near optimum moisture where fill materials are predominantly sandy). Fill areas with in-place density tests showing moisture contents less than those recommended will require additional moisture conditioning prior to placing additional fill.

6.6 Rammed Aggregate Piers

6.6.1 The development can be supported by spread footings if used in conjunction with Rammed Aggregate Piers (RAPs). RAPs such as Geopier® Foundation Systems are designed and installed by specialty ground improvement contractors. The RAP system is based on soil improvement that consists of installing densified, aggregate columns within drilled shafts. Drilling depths depend on specific site soil conditions but are generally on the order of 10 to 20 feet. Deeper elements may be required due to site liquefaction potential. Shaft diameters are commonly 30 inches. The system increases density and lateral stress in the surrounding soil and claims improvement in bearing capacity and settlement potential; thus, allowing the use of conventional shallow foundations over the RAP elements. RAPs typically allow the use of increased allowable bearing pressures for foundation design and result in estimated post-construction total and differential settlements of less than 1 inch and ½ inch, respectively.

6.6.2 RAP elements are constructed by drilling shafts that are subsequently backfilled with Class 2 aggregate base (AB) in approximate 1-foot lifts. An excavator equipped with a special ramming attachment is used to compact each lift of aggregate. Drill spoils are commonly reused as fill material or exported for offsite disposal. Soil and groundwater conditions at the site may require the use of temporary casing during RAP construction.

6.6.3 If the RAP system is selected for structural support, the RAP specialty contractor would provide a complete design-build submittal with design recommendations, engineered plans and specifications. Geocon will need to perform a geotechnical review of the RAP design.

6.6.4 Geocon should monitor RAP construction. Our Quality Assurance (QA) services will supplement the contractor internal Quality Control (QC) program. Together the QA/QC program will monitor drill depths, shaft length, average lift thicknesses, installation procedures, aggregate quality, and densification of lifts. The allowable vertical capacities should be verified by full-scale modulus and uplift load tests performed on RAP elements. The contractor QC program should document each RAP element installed, which will be reviewed by Geocon.

6.7 Shallow Foundation Recommendations

6.7.1 Ancillary site structures such as short retaining walls, screen walls, or trash enclosures may utilize conventional foundations consisting of continuous strip footings founded in competent native alluvial materials or properly compacted fill. Shallow foundations may be utilized for the hotel structure if supported by RAP elements. The following recommendations assume that soils within 5 feet of finish grade will consist of moderately expansive materials (Expansion Index less than 90).

6.7.2 It is recommended that conventional continuous footings have a minimum embedment depth of 18 inches below lowest adjacent pad grade. The footings should be at least 12 inches wide.

6.7.3 Footings proportioned as recommended may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf). The allowable bearing pressure is for dead + live loads and may be increased by up to one-third for transient loads due to wind or seismic forces.

6.7.4 The allowable passive pressure used to resist lateral movement of the footings may be assumed to be equal to a fluid weighing 300 pounds per cubic foot (pcf). The allowable coefficient of friction to resist sliding is 0.30 for concrete against soil. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

6.7.5 Minimum reinforcement for continuous footings should consist of four No. 4 steel reinforcing bars; two placed near the top of the footing and two near the bottom.

6.7.6 The foundation dimensions and minimum reinforcement recommendations presented herein are based upon soil conditions only and are not intended to be used in lieu of those required for structural purposes.

6.7.7 Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 plane extending out and down from the bottom edge of the footing.

6.7.8 The foundation subgrade should be sprinkled as necessary to maintain a moist condition without significant shrinkage cracks as would be expected in any concrete placement. Our representative should observe all footing excavations prior to placing reinforcing steel.

6.8 Temporary Excavations

6.8.1 The native alluvium can be considered a Type B soil in accordance with OSHA guidelines. Where free water, sandy or cohesionless soils or undocumented fills are encountered the materials should be downgraded to Type C. The contractor should have a “competent person” as defined by OSHA evaluate all excavations. All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring.

6.8.2 It is the contractor’s responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements.

6.9 Underground Utilities

6.9.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than six inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding eight inches and should be compacted to at least 90% relative compaction at least 2% above optimum moisture content (near optimum where backfill materials are predominantly sands and gravels).

6.9.2 Bedding and pipe zone backfill typically extends from the bottom of the trench excavations to a minimum of 6 inches above the crown of the pipe. Pipe bedding material should consist of crushed aggregate, clean sand or similar open-graded material. Proposed bedding and pipe zone materials should be reviewed by Geocon prior to construction; open-graded materials such as ¾ inch drain rock may require wrapping with filter fabric to mitigate the potential for piping. Pipe bedding and backfill should also conform to the requirements of the governing utility agency.

6.10 Concrete Slabs-on-Grade

6.10.1 Exterior concrete slabs-on-grade subject to vehicle loading are considered pavements should be designed in accordance with the recommendations in Section 6.12 of this report.

6.10.2 Slabs-on-grade should be underlain by at least 12 inches of low-expansive fill meeting the requirements of Section 6.4.2 to reduce the potential for slab distress due shrink/swell in the native expansive soils.

6.10.3 Concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 5 inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 24 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

6.10.4 Interior slabs or slabs in areas where moisture would be objectionable should be underlain by 3 inches of ½-inch or ¾-inch crushed rock with no more than 5% passing the No. 200 sieve to serve as a capillary break.

6.10.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 24 inches on center in both horizontal directions, positioned near the slab midpoint. We recommend that at least 6 inches of Class 2 Aggregate Base (AB) compacted to at least 95% relative compaction be used below exterior concrete slabs. Prior to placing AB, the subgrade should be moisture conditioned to at least 2% over optimum and properly compacted to at least 90% relative compaction.

6.10.6 In lieu of specific recommendations from the structural or civil engineer, we recommend that crack control joints be spaced at intervals not greater than 8 feet for 4-inch-thick slabs (10 feet for 5-inch slabs). Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.10.7 The recommendations of this report are intended to reduce the potential for cracking of slabs due to soil movement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to soil movement. This is common for project areas that contain expansive soils since designing to eliminate potential soil movement is cost prohibitive. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.11 Moisture Protection Considerations

6.11.1 A vapor barrier is not required beneath slab-on-grade or mat foundations for geotechnical purposes. Further, the migration of moisture through concrete slabs or moisture otherwise released from slabs is not a geotechnical issue. However, for the convenience of the owner, we are providing the following general suggestions for consideration by the owner, architect, structural engineer, and contractor. The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field. If a vapor barrier is used beneath mat slab foundations, friction between the mat slab and underlying substrate when evaluating lateral loading resistance.

6.11.2 A vapor barrier meeting ASTM E 1745-09 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) should be used. The vapor barrier, if used, should extend to the edges of the slab, and should be sealed at all seams and penetrations.

6.11.3 The concrete water/cement ratio should be as low as possible. The water/cement ratio should not exceed 0.45 for concrete placed directly on the vapor barrier. Midrange plasticizers could be used to facilitate concrete placement and workability.

6.11.4 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

6.12 Pavement Recommendations

6.12.1 The upper 12 inches of pavement subgrade should be scarified, moisture conditioned to at least 2% over optimum and compacted to at least 95% relative compaction. Prior to placing aggregate base, the finished subgrade should be proof-rolled with a laden water truck (or similar equipment with high contact pressure) to verify stability.

6.12.2 Sidewalk, curb, gutter, and driveway encroachments should be designed and constructed in accordance with City of San Jose requirements, as applicable.

6.12.3 We recommend the following asphalt concrete (AC) pavement sections for design to establish subgrade elevations in pavement areas. The project civil engineer should determine the appropriate Traffic Index (TI) based on anticipated traffic conditions. The flexible pavement sections below are based on estimated design TIs and an R-Value of 5 for the subgrade soils. We can provide additional sections based on other TIs if necessary.

TABLE 6.12
FLEXIBLE PAVEMENT SECTION RECOMMENDATIONS

Location	Estimated Traffic Index (TI)	AC Thickness (inches)	Class 2 AB Thickness (inches)
Parking Stalls	4.5	3	8
Driveways	6.0	3 ½	12 ½
Heavy-Duty	7.0	4	15 ½

Note: The recommended flexible pavement sections are based on the following assumptions:

1. AB: Class 2 AB with a minimum R-Value of 78 and meeting the requirements of Section 26 of the latest Caltrans Standard Specifications.
2. AB is compacted to 95% or higher relative compaction at or near optimum moisture content. Prior to placing AB, the subgrade should be proof-rolled with a loaded water truck to verify stability.
3. AC: Asphalt concrete conforming to local agency standards or Section 39 of the latest Caltrans Standard Specifications.

6.12.4 The AC sections in Table 6.12 are final, minimum thicknesses. If staged-pavements are used, the construction bottom AC lift should be at least 2 inches thick. Following construction, the finish top AC lift should be at least 1½ inches thick.

6.12.5 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, we recommend the concrete be a minimum of 6 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. In addition, doweling, reinforcing steel or other load-transfer mechanism should be provided at joints if desired to reduce the potential for vertical offset. The concrete should have a minimum 28-day compressive strength of 3,500 psi. We should evaluate pavements to support heavy truck traffic on a case-by-case basis; supplemental recommendations may be provided.

6.12.6 We recommend that at least 6 inches of Class 2 Aggregate Base be used below rigid concrete pavements. The aggregate base should be compacted to at least 95% relative compaction near optimum moisture content.

6.12.7 In general, we recommend that concrete pavements be designed, constructed and maintained in accordance with industry standards such as those provided by the American Concrete Pavement Association.

6.12.8 Crack control joints should be spaced at intervals not greater than 12 feet for 6-inch slabs and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.12.9 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 6 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving. Alternatives such as plastic moisture cut-offs or modified drop-inlets may also be considered in lieu of deepened curbs.

6.13 Retaining Wall Design

6.13.1 Lateral earth pressures may be used in the design of retaining walls and buried structures. Lateral earth pressures against these facilities may be assumed to be equal to the pressure exerted by an equivalent fluid. The unit weight of the equivalent fluid depends on the design conditions. Table 6.13 summarizes the weights of the equivalent fluid based on the different design conditions.

TABLE 6.13
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active	45 pcf
At-Rest	65 pcf

6.13.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.001H (where H is the height of the wall). Walls restrained from movement such as basement walls should be designed using the at-rest case. The above soil pressures assume level backfill under drained conditions within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall and no surcharges within that same area. Where restrained basement walls will be undrained, an at-rest equivalent fluid density of 100 pounds per cubic foot (pcf) should be used for retaining wall design.

6.13.3 Unless project-specific loading information is provided by the structural engineer, where vehicle loads are expected atop the wall backfill, an additional uniform surcharge pressure equivalent to 2 feet of backfill soil should be used for design. Where the vehicle loading will be limited to passenger cars, the additional uniform surcharge equivalent may be reduced to 1 foot of backfill soil.

6.13.4 Retaining walls greater than 2 feet tall (retained height) should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. Positive drainage for retaining walls should consist of a vertical layer of permeable material positioned between the retaining wall and the soil backfill. The permeable material may be composed of a composite drainage geosynthetic or a natural permeable material such as crushed gravel at least 12 inches thick and capped with at least 12 inches of native soil. A geosynthetic filter fabric should be placed between the gravel and the soil backfill. Provisions for removal of collected water should be provided for either system by installing a perforated drainage pipe along the bottom of the permeable material which leads to suitable drainage facilities.

6.13.5 We recommend that all retaining wall designs be reviewed by Geocon to confirm the incorporation of the recommendations provided herein. In particular, potential surcharges from adjacent structures and other improvements should be reviewed by Geocon.

6.14 Surface Drainage

6.14.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change to important engineering properties. Proper drainage should be maintained at all times.

6.14.2 All site drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundations or retaining walls. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structures should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers not permitted onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed or properly drained to prevent moisture intrusion into the materials providing foundation support. Landscape irrigation within five feet of the building perimeter footings should be kept to a minimum to just support vegetative life.

6.14.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond. Final soil grade should slope a minimum of 2% away from structures.

6.14.4 We recommend implemented measures to reduce infiltrating surface water near buildings and slabs-on-grade. Such measures may include:

- Selecting drought-tolerant plants that require little or no irrigation, especially within 5 feet of buildings, slabs-on-grade, or pavements.
- Using drip irrigation or low-output sprinklers.
- Using automatic timers for irrigation systems.

- Appropriately spaced area drains.
- Hard-piping roof downspouts to appropriate collection facilities.

7. FURTHER GEOTECHNICAL SERVICES

7.1 Plan and Specification Review

7.1.1 We should review project plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

7.2 Testing and Observation Services

7.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase and provide compaction testing and observation services and foundation observations throughout the project. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others interpretation of our recommendations, and therefore the future performance of the project.

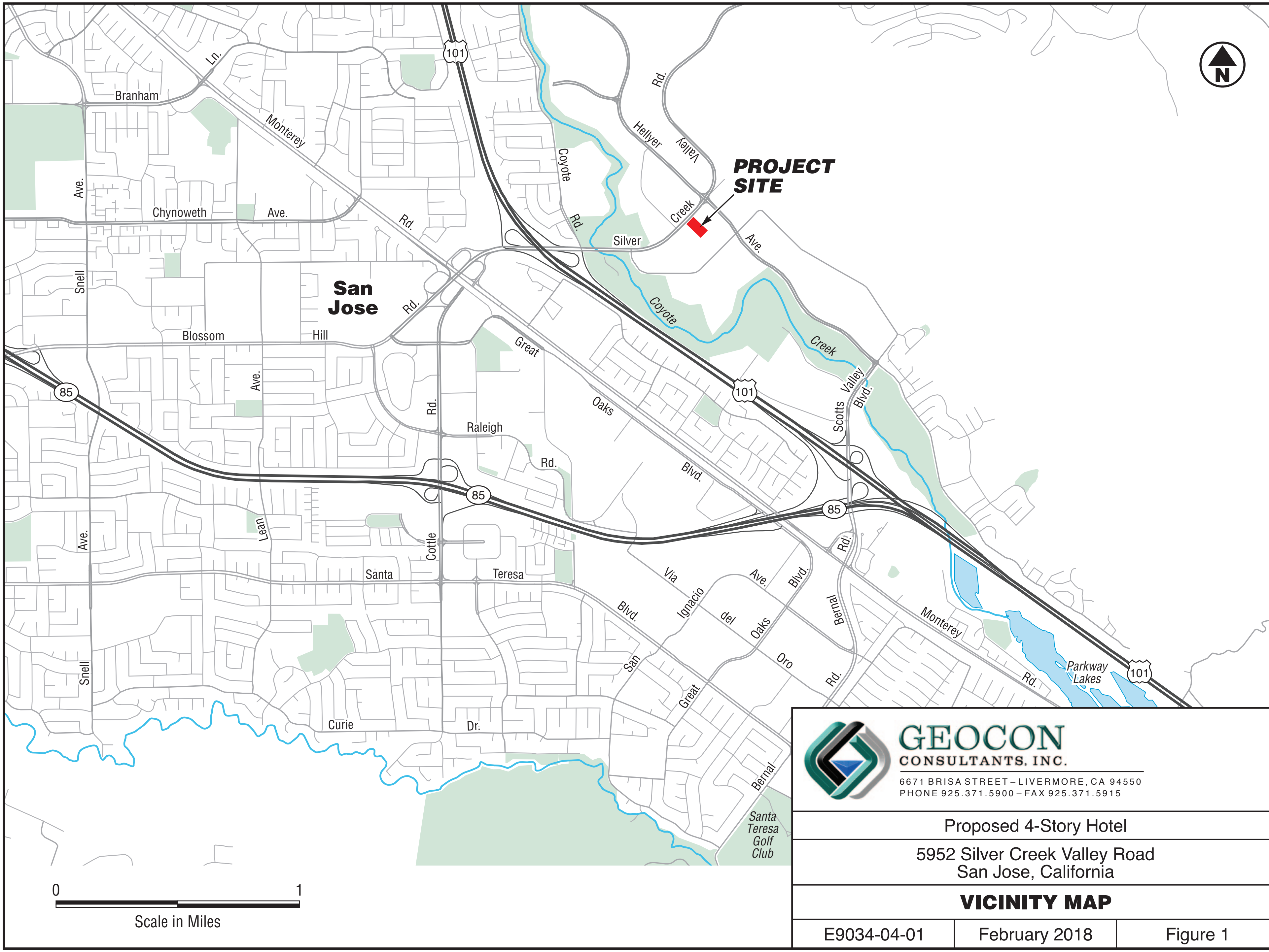
LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Consultants, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the geotechnical scope of services provided by Geocon Consultants, Inc.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the site area at this time. No warranty is provided, express or implied.



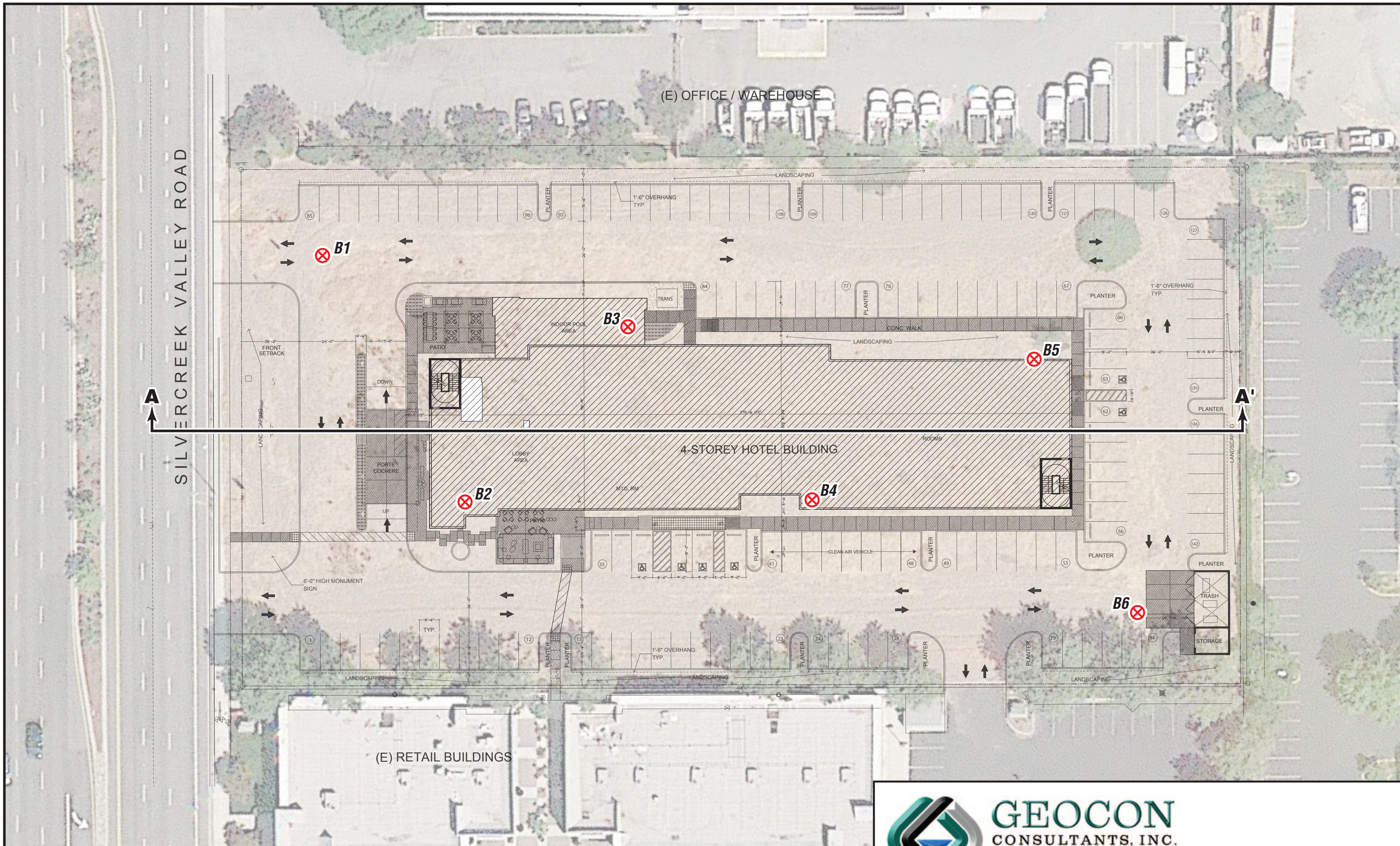
GEOCON
CONSULTANTS, INC.
6671 BRISA STREET – LIVERMORE, CA 94550
PHONE 925.371.5900 – FAX 925.371.5915

Proposed 4-Story Hotel

5952 Silver Creek Valley Road
San Jose, California

VICINITY MAP

E9034-04-01	February 2018	Figure 1



LEGEND:

B6 ⊗ Approximate Boring Location

A — A' Approximate Cross-Section Location

0 60

Scale in Feet

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Proposed 4-Story Hotel

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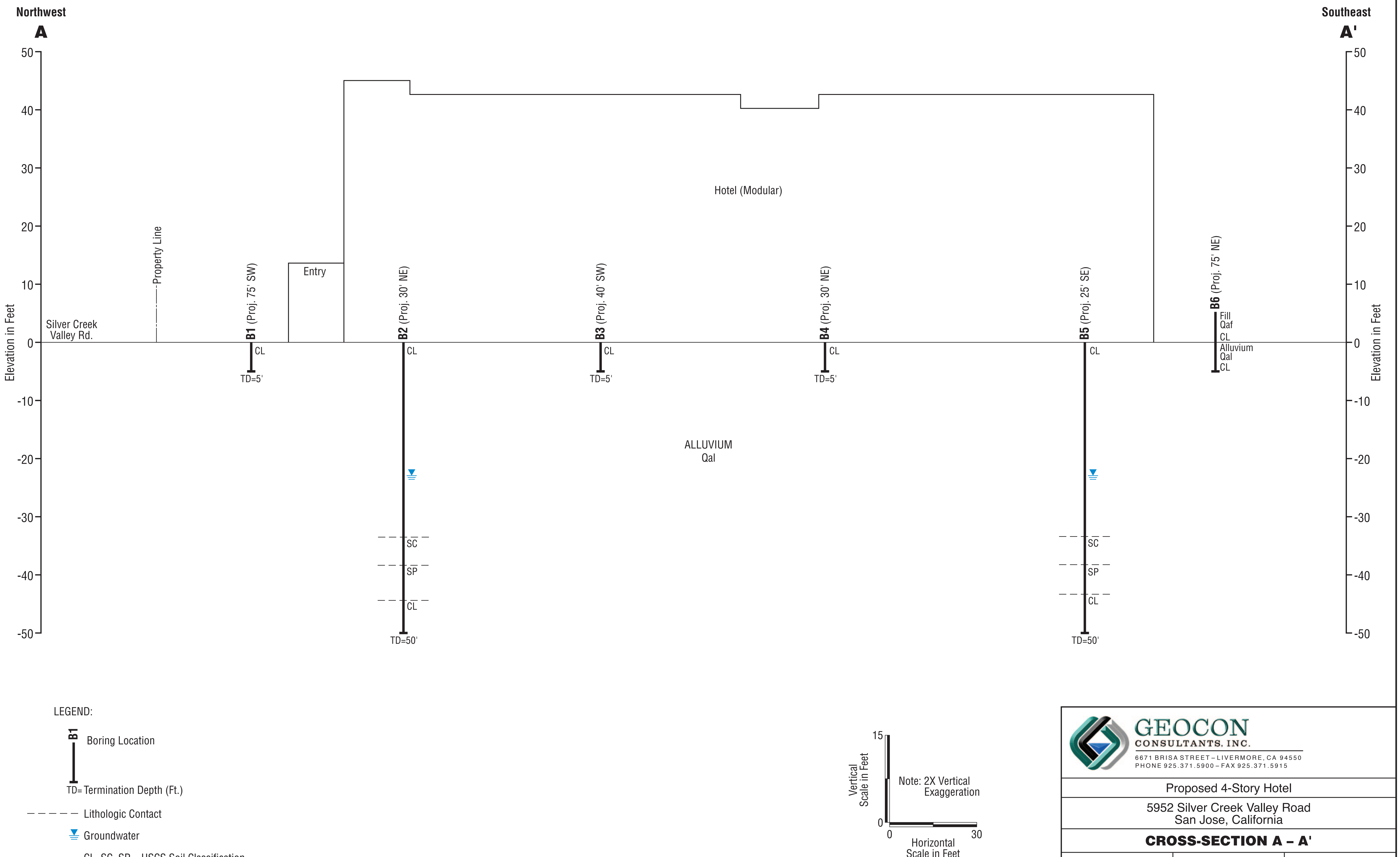
San Jose, California

SITE PLAN

E9034-04-01	February 2018	Figure 2



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Proposed 4-Story Hotel

5952 Silver Creek Valley Road
San Jose, California

CROSS-SECTION A – A'

E9034-04-01	February 2018	Figure 3

APPENDIX A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation included a site visit, subsurface exploration, and soil sampling. The locations of our exploratory borings are shown on the Site Plan, Figure 2. Soil boring logs for our exploration are presented as figures following the text in this appendix. The borings were located by pacing from existing reference points. Therefore, the exploration locations shown on Figure 2 are approximate.

Our field exploration included the advancement of six exploratory soil borings. Our borings were performed on December 27, 2017 using a truck-mounted Mobile B-53 drill rig equipped with 8-inch hollow-stem augers. Sampling in the borings was accomplished using a down-hole wire-line 140-pound hammer with a 30-inch drop. Samples were obtained with a 3-inch outside-diameter (OD), split spoon (California Modified) sampler, and a 2-inch OD, Standard Penetration Test (SPT) sampler. The number of blows required to drive the sampler the last 12 inches (or fraction thereof) of the 18-inch sampling interval were recorded on the boring logs. The blow counts shown on the boring logs should not be interpreted as standard SPT "N" values; corrections have not been applied.

Subsurface conditions encountered in the exploratory boring were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488). This system uses the Unified Soil Classification System (USCS) for soil designations. The log depicts soil and geologic conditions encountered and depths at which samples were obtained. The log also includes our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, drill rig penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing.

Upon completion, our boreholes were backfilled per Santa Clara Valley Water District requirements.

PROJECT NO. **E9034-04-01** PROJECT NAME: **Raka Hotel San Jose**

BORING B1

ELEV. (MSL.) ______ DATE COMPLETED **12/27/2017**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B1-0-5			CL	ALLUVIUM Stiff, damp to moist, brown, CLAY with few silts and (f) sands			
1								
2								
	B1-2.5				-pp>4½	22		
3	B1-3						100.1	12.6
4	B1-4				-pp>4½	18		
	B1-4.5							
5					END OF BORING AT APPROXIMATELY 5 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS			

Figure A2, Log of Boring B1, page 1 of 1

GEOCON BORING LOG E9034-04-01 BORING LOGS.GPJ 01/29/18

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B2

ELEV. (MSL.) ______ DATE COMPLETED **12/27/2017**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	ALLUVIUM Stiff, damp to moist, brown, CLAY with few silts and (f) sands -roots			
3	B2-2.5-3 B2-3				-pp>4½	20		
4	B2-4-4.5 B2-4.5				-very stiff -no roots -pp>4½	25	99.6	13.9
9	B2-9-9.5 B2-9.5				-moist, less silt -pp=4½	31		
10					-more sand			
14	B2-14-14.5 B2-14.5				-stiff, light brown -pp=2½-3¼	21	102.1	24.3
19	B2-19-19.5 B2-19.5				-very stiff, gray-brown -pp=3½-4	34		

Figure A3, Log of Boring B2, page 1 of 3

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

BORING B2

ELEV. (MSL.)		DATE COMPLETED	12/27/2017
ENG./GEO.	JBM	DRILLER	EGI
EQUIPMENT	Mobile B53 w/ 8-inch HSA	HAMMER TYPE	Downhole-Wireline

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
20								
21								
22								
23			▼					
24	B2-24-24.5 B2-24.5				-stiff, moist to wet -pp=1-1¾	18		
25								
26								
27								
28								
29	B2-29-29.5 B2-29.5				-wet	24		
30								
31								
32								
33								
34	B2-34-34.5 B2-34.5			SC	Medium dense, moist, gray-brown, Clayey (f-m) SAND	45	110.3	18.1
35								
36								
37								
38								
39	B2-39-40			SP	Medium dense, wet, gray, (f-c) SAND with trace clays and (f) gravels	17		

Figure A3, Log of Boring B2, page 2 of 3

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B2

ELEV. (MSL.) ______ DATE COMPLETED **12/27/2017**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
40								
41								
42								
43								
44	B2-44-45			CL	Hard, damp, gray, CLAY with trace (f) sand -pp>4½	45		
45								
46								
47								
48								
49	B2-49-49.5 B2-49.5				-pp>4½	94/12"		
50					END OF BORING AT APPROXIMATELY 50 FEET GROUNDWATER INITIALLY ENCOUNTERED AT APPROXIMATELY 23 FEET BACKFILLED WITH GROUT			

Figure A3, Log of Boring B2, page 3 of 3

GEOCON

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

PROJECT NO. **E9034-04-01** PROJECT NAME: **Raka Hotel San Jose**

BORING B3

ELEV. (MSL.) ______ DATE COMPLETED **12/27/2017**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	ALLUVIUM Very stiff, damp, brown, CLAY with few silts and (f) sands			
1								
2								
	B3-2.5					27		
3	B3-3				-pp>4½			
4	B3-4				-stiff	24		
	B3-4.5				-pp>4½		103.3	17.3
5					END OF BORING AT APPROXIMATELY 5 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS			

Figure A4, Log of Boring B3, page 1 of 1

GEOCON BORING LOG E9034-04-01 BORING LOGS.GPJ 01/29/18

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B4

ELEV. (MSL.)		DATE COMPLETED	12/27/2017
ENG./GEO.	JBM	DRILLER	EGI
EQUIPMENT	Mobile B53 w/ 8-inch HSA	HAMMER TYPE	Downhole-Wireline

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	ALLUVIUM Very stiff, damp, brown, CLAY with few silts and (f) sands			
1								
2								
	B4-2.5					29		
3	B4-3				-pp>4½		94.5	16.6
4	B4-4					24		
	B4-4.5				-stiff -pp>4½			
5					END OF BORING AT APPROXIMATELY 5 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS			

Figure A5, Log of Boring B4, page 1 of 1

GEOCON BORING LOG E9034-04-01 BORING LOGS.GPJ 01/29/18

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **E9034-04-01** PROJECT NAME: **Raka Hotel San Jose**

BORING B5

ELEV. (MSL.) ____________ DATE COMPLETED **12/27/2017**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B5-0-5			CL	ALLUVIUM Stiff, damp to moist, brown, Silty CLAY with few (f) sands			
3	B5-2.5-3 B5-3				-pp=4-4½	18		
4	B5-4-4.5 B5-4.5				-pp>4½	15	108.1	18.1
9	B5-9-9.5 B5-9.5				-very stiff -more sand, less silt -pp=4-4½	28	110.6	16.1
14	B5-14-14.5 B5-14.5				-stiff, less sand -pp=4-4½+	24	106.8	18.8
19	B5-19-19.5 B5-19.5				-very stiff, more sand	26		

Figure A6, Log of Boring B5, page 1 of 3

GEOCON BORING LOG E9034-04-01 BORING LOGS.GPJ 01/29/18

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

PROJECT NO. **E9034-04-01** PROJECT NAME: **Raka Hotel San Jose**

BORING B5

ELEV. (MSL.) ________ DATE COMPLETED **12/27/2017**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
20								
21								
22								
23			▼					
24	B5-24-25				-stiff, less sand	12		
25								
26								
27								
28								
29	B5-29-30				-very stiff, wet, more sand	18		
30								
31								
32								
33								
34	B5-34-35			SC	Very dense, wet, gray and varicolored, Clayey (f-c) SAND	65		
35								
36								
37								
38								
39	B5-39-40			SP	Very dense, wet, gray, (f-c) SAND with trace (f) subrounded to rounded gravels	51		

Figure A6, Log of Boring B5, page 2 of 3

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

BORING B5

ELEV. (MSL.) ____ DATE COMPLETED **12/27/2017**
ENG./GEO. **JBM** DRILLER **EGI**
EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
40								
41								
42								
43								
44	B5-44-45			CL	Hard, damp, white to light gray, CLAY with silt -pp>4½	83/11"		
45								
46								
47								
48								
49	B5-49.5				-pp>4½	50/5"		
					END OF BORING AT APPROXIMATELY 50 FEET GROUNDWATER INITIALLY ENCOUNTERED AT APPROXIMATELY 23 FEET BACKFILLED WITH GROUT			

Figure A6, Log of Boring B5, page 3 of 3

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

BORING B6

ELEV. (MSL.) ________ DATE COMPLETED 12/27/2017

ENG./GEO. JBM DRILLER EGI

EQUIPMENT Mobile B53 w/ 8-inch HSA HAMMER TYPE Downhole-Wireline

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B6-0-5			CL	FILL Very stiff, damp, brown, CLAY with few (f-m) subrounded to rounded gravels, (f) sands, and silts			
1								
2								
3	B6-2.5 B6-3				-pp>4½	29		
4	B6-4 B6-4.5			CL	ALLUVIUM Stiff, damp, brown, CLAY with few (f) sands and silts -pp>4½	24		
5								
6								
7								
8								
9	B6-9 B6-9.5				-pp>4½	17		
10					END OF BORING AT APPROXIMATELY 10 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS			

Figure A7, Log of Boring B6, page 1 of 1

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

APPENDIX B

APPENDIX B
LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for in-situ dry density and moisture content, grain size distribution, Atterberg Limits and unconfined compressive strength. The results of our testing are summarized in tabular format below and the following figures. In-situ dry density and/or moisture content test results are included on the boring logs in Appendix A.

TABLE B-I
SUMMARY OF LABORATORY ATTERBERG LIMITS TEST RESULTS
ASTM D 4318

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index
B4-2.5	40	20	20

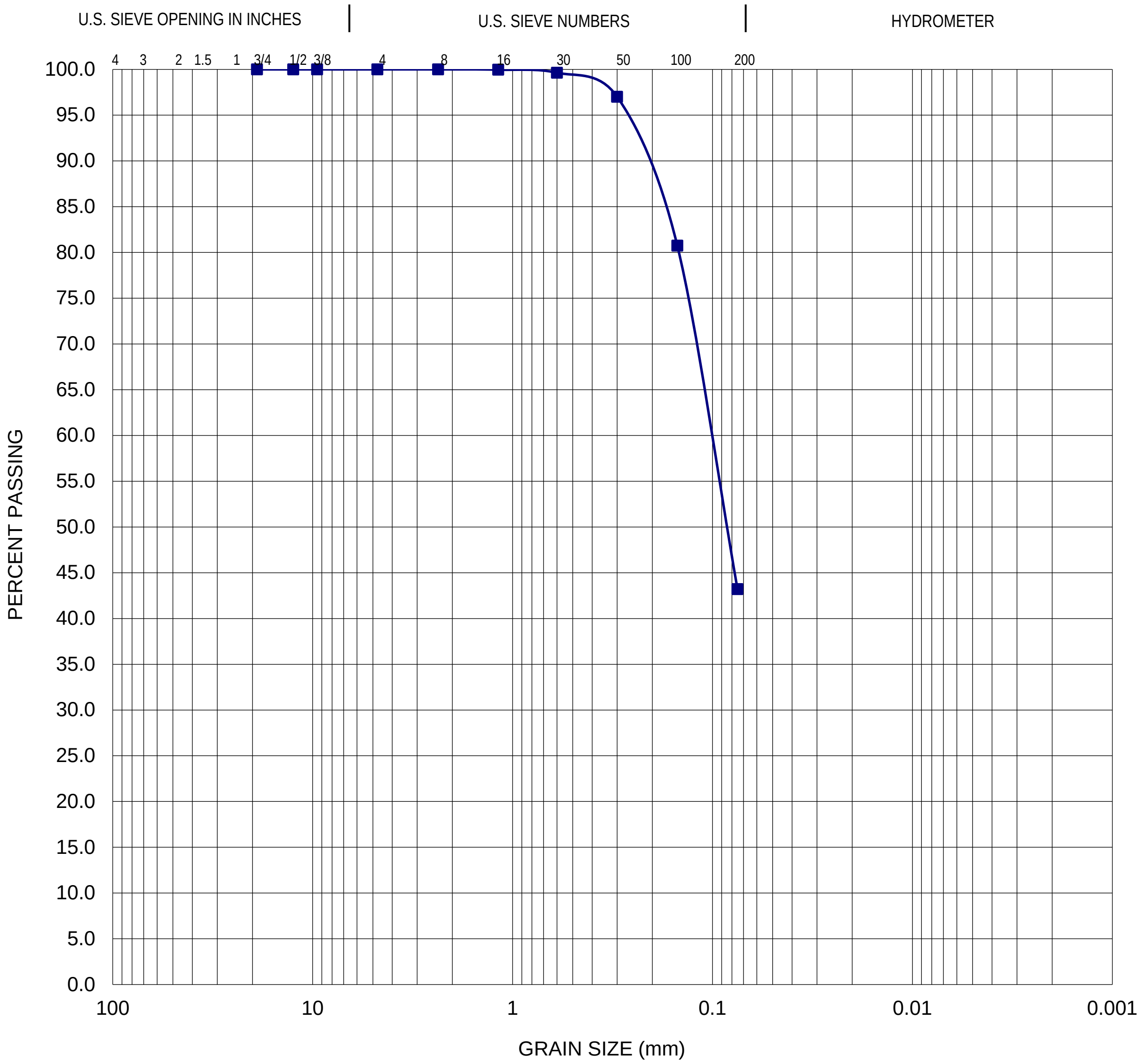
TABLE B-II
SUMMARY OF LABORATORY GRAIN SIZE ANALYSIS – NO. 200 WASH
ASTM D1140

Boring No.	Sample Depth (feet)	Fraction Passing No. 200 Sieve (%)
B2	24-24.5	66
B2	29-29.5	54
B5	19.5	53
B6	24-25	91

TABLE B-III
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content		Dry Density* (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B1-0-5	11.8	26.7	104.2	81

*before saturation



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring: B2
Depth To Sample: 34'-34.5'
Sieve Date: 1/17/17
Tested and Computed by: AC

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100	100	100	100	100	100	100	100	99.6	97.0	80.7	43.2

Geocon Consultants, Inc.
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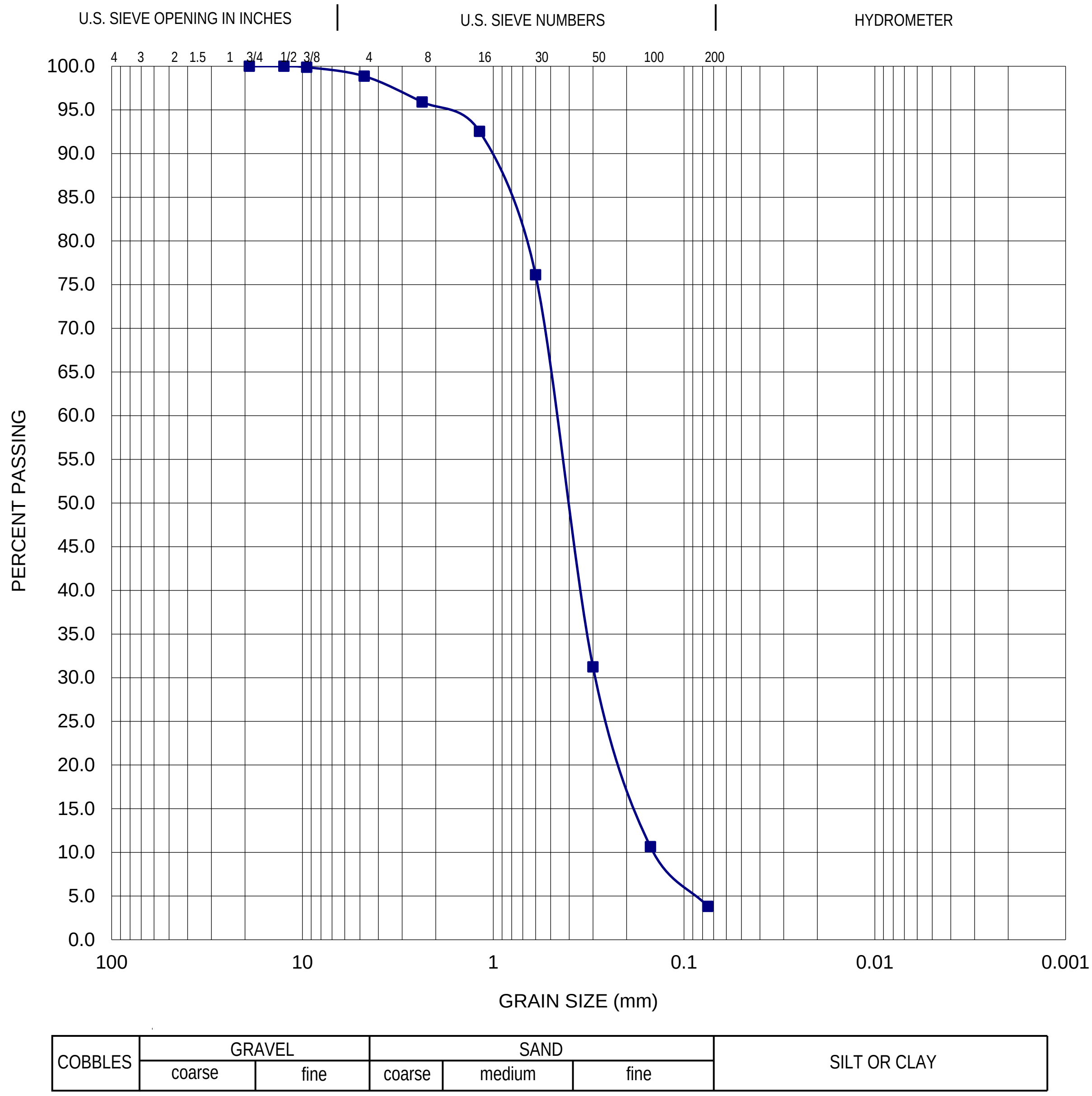
Fax: (925) 371-5915

Particle Size Analysis - ASTM D422
Project: Raka Hotel San Jose GI
Location: San Jose, CA
Project No.: E9034-04-01

Figure B1



Boring: B2

Depth To Sample: 39'-40'

Sieve Date: 1/17/17

Tested and Computed by: AC

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100	100	100	100	99.9	98.9	95.9	92.5	76.1	31.2	10.7	3.8

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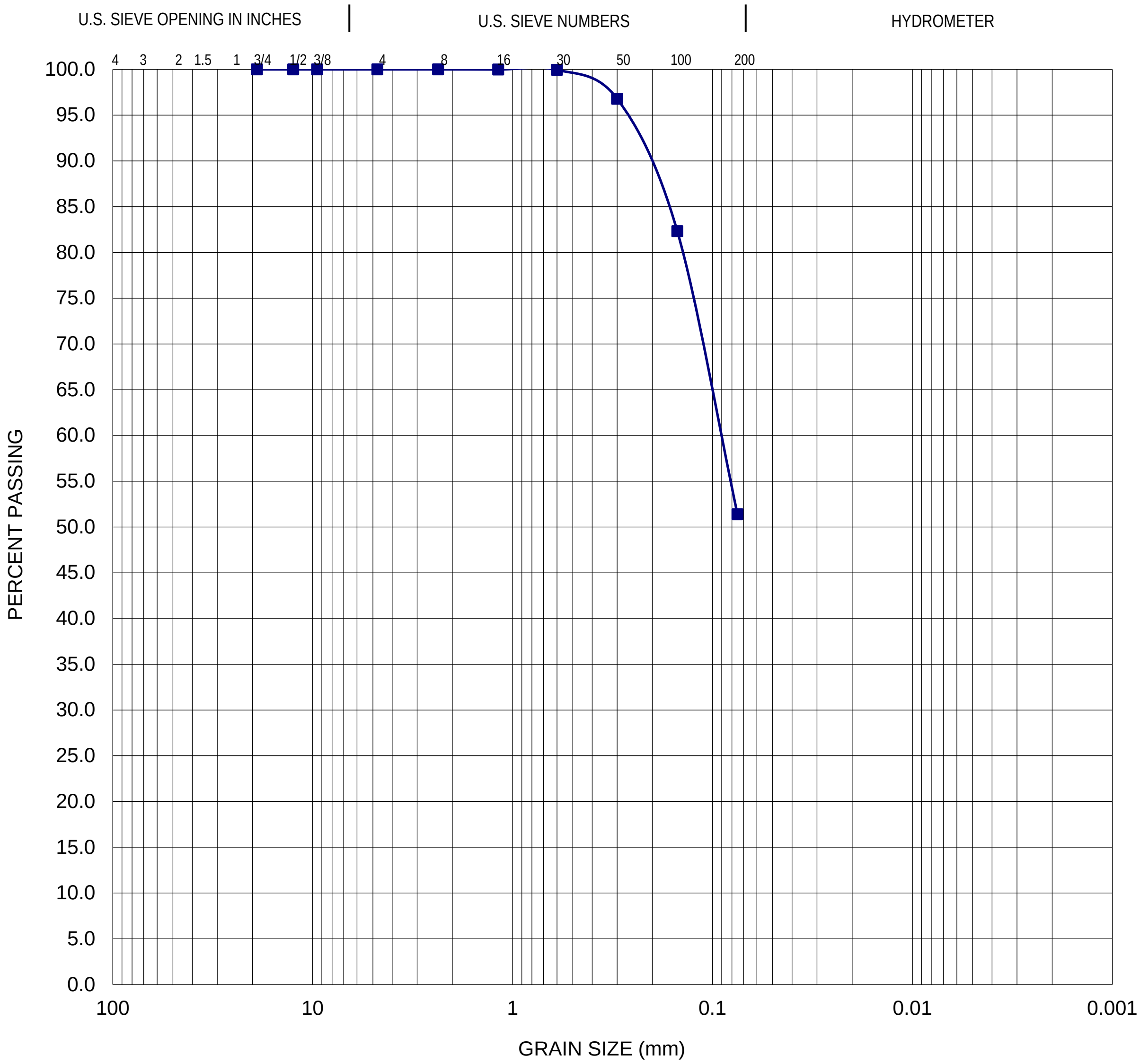
Particle Size Analysis - ASTM D422

Project: Raka Hotel San Jose GI

Location: San Jose, CA

Project No.: E9034-04-01

Figure B2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring: B5

Depth To Sample: 29'-30'

Sieve Date: 1/17/17

Tested and Computed by: AC

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100	100	100	100	100	100	100	100	99.9	96.8	82.3	51.4

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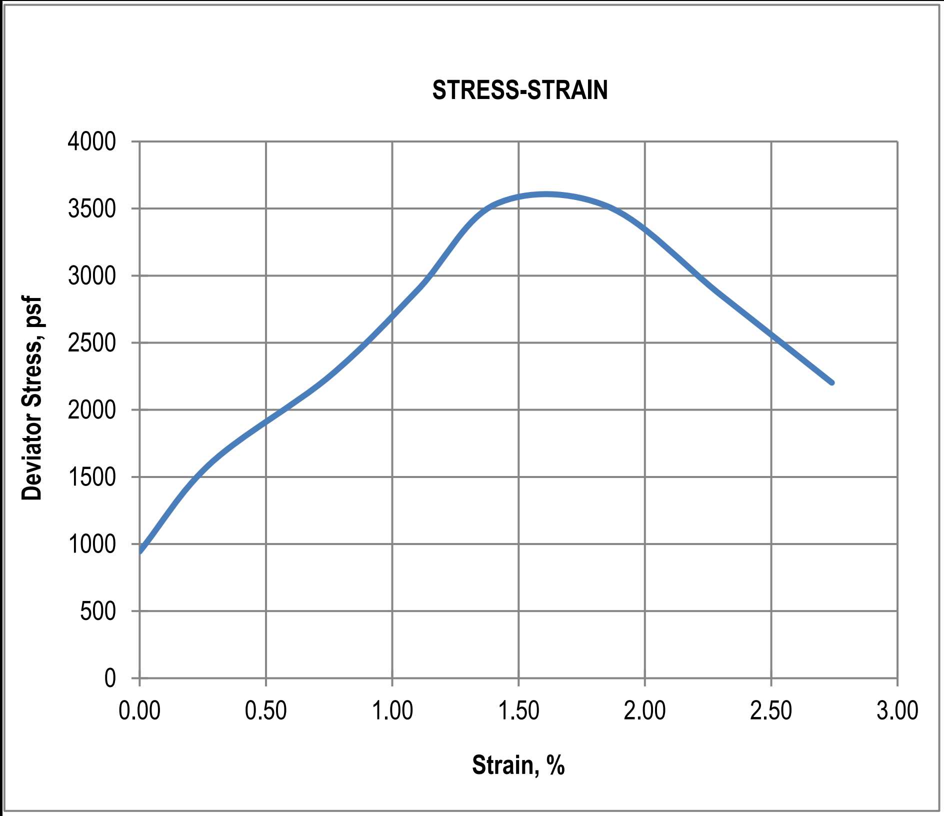
Particle Size Analysis - ASTM D422

Project: Raka Hotel San Jose GI

Location: San Jose, CA

Project No.: E9034-04-01

Figure B3



Failure Photo

Sample Description	
Boring Number	B2
Sample Depth (feet)	4.5'
Material Description	Medium brown CLAY with silt
Initial Conditions at Start of Test	
Height (inch) average of 3	5.62
Diameter (inch) average of 3	2.40
Moisture Content (%)	13.9
Dry Density (pcf)	99.6
Estimated Specific Gravity	2.7
Saturation (%)	54.2
Shear Test Conditions	
Strain Rate (%/min)	0.6852
Major Principal Stress at Failure (psf)	3530
Strain at Failure (%)	1.4
Test Results	
Unconfined Compressive Strength (tons/ft^2)	1.8
Unconfined Compressive Strength (lbs/ft^2)	3534
Shear Strength (tons/ft^2)	0.9
Shear Strength (lbs/ft^2)	1767

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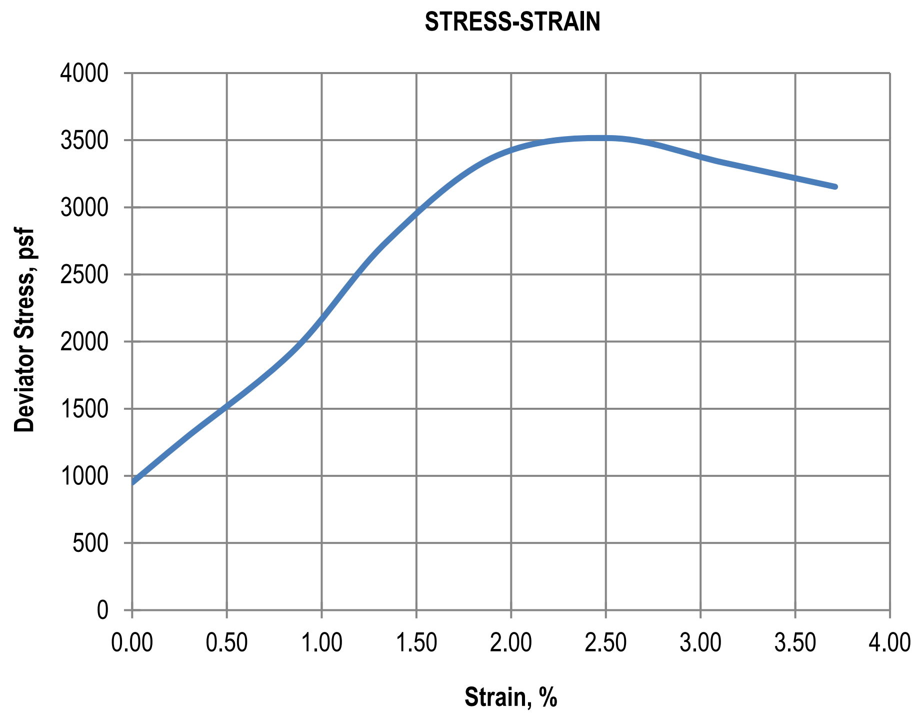
Unconfined Compressive Strength (ASTM D2166)

Project: Raka Hotel San Jose GI
Location: San Jose, CA
Proj. No.: E9034-04-01

Figure B4



Failure Photo

Sample Description	
Boring Number	B4
Sample Depth (feet)	3'
Material Description	Medium brown CLAYEY silt
Initial Conditions at Start of Test	
Height (inch) average of 3	3.88
Diameter (inch) average of 3	2.39
Moisture Content (%)	16.6
Dry Density (pcf)	94.5
Estimated Specific Gravity	2.7
Saturation (%)	57.3
Shear Test Conditions	
Strain Rate (%/min)	0.9285
Major Principal Stress at Failure (psf)	3520
Strain at Failure (%)	2.5
Test Results	
Unconfined Compressive Strength (tons/ft^2)	1.8
Unconfined Compressive Strength (lbs/ft^2)	3515
Shear Strength (tons/ft^2)	0.9
Shear Strength (lbs/ft^2)	1758

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Unconfined Compressive Strength (ASTM D2166)

Project: Raka Hotel San Jose GI
Location: San Jose, CA
Proj. No.: E9034-04-01

Figure B5

APPENDIX C

APPENDIX C
SELECTED OUTPUT – LIQUEFACTION ANALYSIS

Liquefaction Analysis Using SPT

Project Name:	Raka Hotel San Jose
Project Number:	E9034-04-01
Boring:	B2

a_{max}/g	0.78
Magnitude	6.60
Groundwater Depth, Ft	16.0
Reference Pressure, p_a	2000
Unit Weight of Water	62.4
Soil Unit Weight, pcf	120

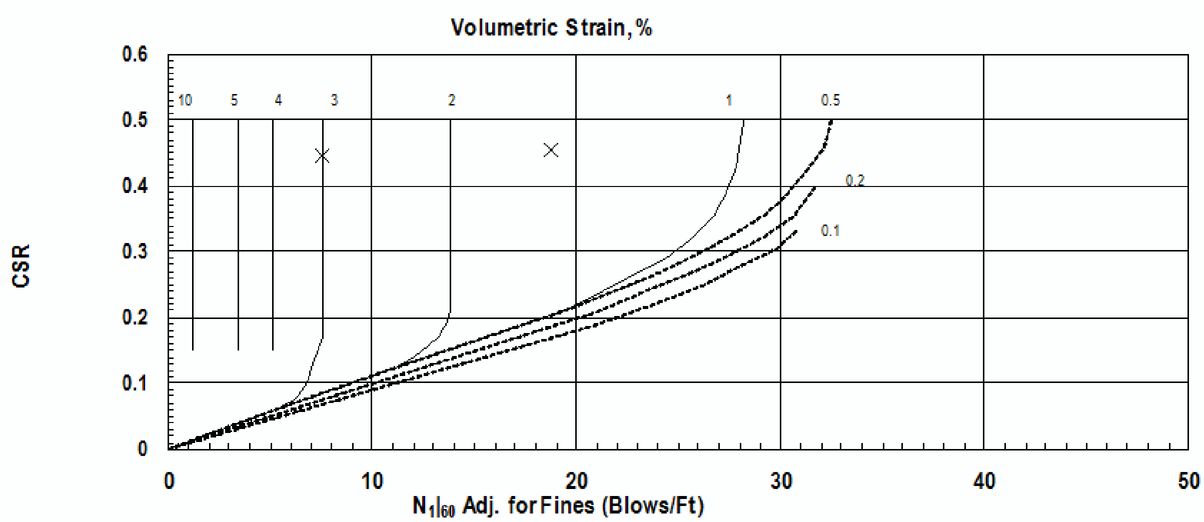
Include Ks (Y/N)	Y
Use NCEER $CRR_{7.5}$ (1) or Rauch $CRR_{7.5}$ (2)	1
Minimum Factor of Safety for Liquefaction	1

MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$ — From Graph

Depth, ft	$N_1\|_{60}$	Fines Content, FC (%)	$N_1\|_{60}$, Adj. for Fines	s, psf	s', psf	r_d	K_s	NCEER $CRR_{7.5}$	RAUCH $CRR_{7.5}$	CSR M=7.5	Liquefaction Potential	Factor of Safety	Volumetric Strain, %	Settlement, in.
0														
10	100	60	125.0	1200.0	1200.0	0.98	1.00	0.800	0.800	0.358	Above GWT	2.235		
20	100	60	125.0	2400.0	2150.4	0.96	0.99	0.800	0.800	0.390	NL	2.049		
33.5	100	60	125.0	4020.0	2928.0	0.90	0.93	0.800	0.800	0.452	NL	1.770		
38.5	11.5	43	18.8	4620.0	3216.0	0.86	0.91	0.186	0.201	0.454	LIQUEFIABLE	0.410	1.7	1.02
44	7.5	4	7.5	5280.0	3532.8	0.81	0.89	0.076	0.092	0.444	LIQUEFIABLE	0.170	3	1.98
50	100	95	125.0	6000.0	3878.4	0.75	0.87	0.800	0.800	0.426	NL	1.880		

NL: Non-Liquefiable

Total Settlement: 3.00



Boring B2 Blow Count Standardization

Sample Midpoint (ft)	Layer Depth (ft)	Blow Count (MC)	Blow Count (SPT)	Soil Unit Weight (PCF)	Depth to Water (ft)	Rod Length (ft)	Overburden (PSF)	C_N	C_E	C_B	C_R	C_S	$N_{1\|60}$
34.5	33.5-38.5	45	28.125	120	23	15	3422.4	0.764	0.63	1	0.85	1	11.5
39.5	38.5-44	--	17	120	23	20	3710.4	0.734	0.63	1	0.95	1	7.5

C_E per driller. Hollow-stem Auger I.D. of 3¾ inches. SPT sampler not designed to hold a liner.

Liquefaction Analysis Using SPT

Project Name:	Raka Hotel San Jose
Project Number:	E9034-04-01
Boring:	B5

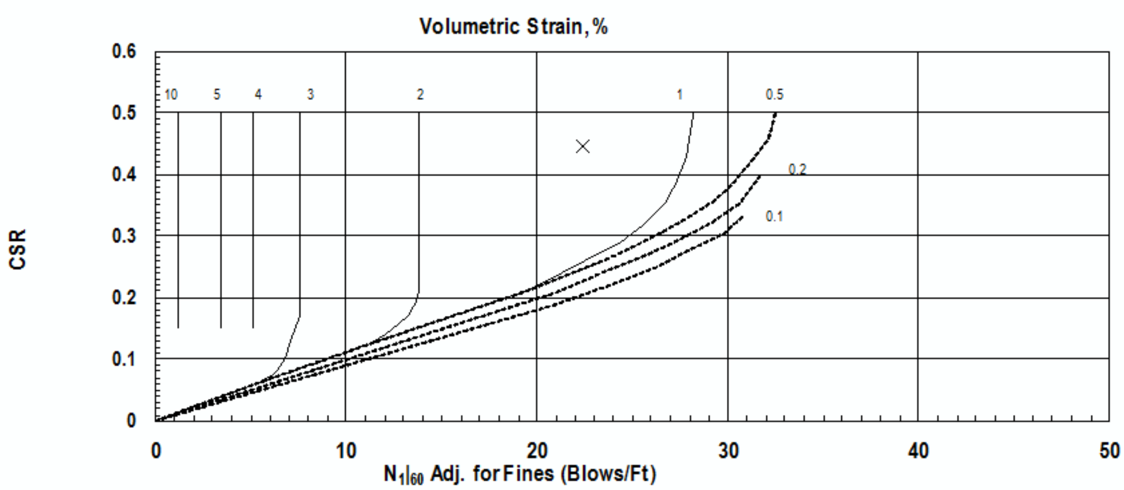
a_{max}/g	0.78	Include Ks (Y/N)	Y
Magnitude	6.60	Use NCEER $CRR_{7.5}$ (1) or Rauch $CRR_{7.5}$ (2)	1
Groundwater Depth, Ft	16.0	Minimum Factor of Safety for Liquefaction	1
Reference Pressure, p_a	2000		
Unit Weight of Water	62.4		
Soil Unit Weight, pcf	120		

MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$ From Graph

Depth, ft	$N_1\|_{60}$	Fines Content, FC (%)	$N_1\|_{60}$, Adj. for Fines	s, psf	s', psf	r_d	K_s	NCEER $CRR_{7.5}$	RAUCH $CRR_{7.5}$	CSR M=7.5	Liquefaction Potential	Factor of Safety	Volumetric Strain, %	Settlement, in.
0														
10	100	60	125.0	1200.0	1200.0	0.98	1.00	0.800	0.800	0.358	Above GWT	2.235		
20	100	60	125.0	2400.0	2150.4	0.96	0.99	0.800	0.800	0.390	NL	2.049		
33.5	100	60	125.0	4020.0	2928.0	0.90	0.93	0.800	0.800	0.452	NL	1.770		
38.5	26.6	43	36.9	4620.0	3216.0	0.86	0.91	0.800	0.800	0.454	NL	1.764		
44	22.4	4	22.4	5280.0	3532.8	0.81	0.89	0.220	0.248	0.444	LIQUEFIABLE	0.495	1.4	0.924
50	100	95	125.0	6000.0	3878.4	0.75	0.87	0.800	0.800	0.426	NL	1.880		

NL: Non-Liquefiable

Total Settlement: 0.92



Boring B5 Blow Count Standardization													
Sample Midpoint (ft)	Layer Depth (ft)	Blow Count (MC)	Blow Count (SPT)	Soil Unit Weight (PCF)	Depth to Water (ft)	Rod Length (ft)	Overburden (PSF)	C_N	C_E	C_B	C_R	C_S	$N_{1\|60}$
34.5	33.5-38.5	--	65	120	23	15	3422.4	0.764	0.63	1	0.85	1	26.6
39.5	38.5-43.5	--	51	120	23	20	3710.4	0.734	0.63	1	0.95	1	22.4

C_E per driller. Hollow-stem Auger I.D. of 3¾ inches. SPT sampler not designed to hold a liner.

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